

Chapter 4 Stability Analysis

4-1. Introduction

a. This chapter presents information on the stability analysis of concrete gravity dams. The basic loading conditions investigated in the design and guidance for the dam profile and layout are discussed. The forces acting on a structure are determined as outlined in Chapter 3.

b. For new projects, the design of a gravity dam is performed through an iterative process involving a preliminary layout of the structure followed by a stability and stress analysis. If the structure fails to meet criteria then the layout is modified and reanalyzed. This process is repeated until an acceptable cross section is attained. The method for conducting the static and dynamic stress analysis is covered in Chapter 5. The reevaluation of existing structures is addressed in Chapter 8.

c. Analysis of the stability and calculation of the stresses are generally conducted at the dam base and at selected planes within the structure. If weak seams or planes exist in the foundation, they should also be analyzed.

4-2. Basic Loading Conditions

a. The following basic loading conditions are generally used in concrete gravity dam designs (see Figure 4-1). Loadings that are not indicated should be included where applicable. Power intake sections should be investigated with emergency bulkheads closed and all water passages empty under usual loads. Load cases used in the stability analysis of powerhouses and power intake sections are covered in EM 1110-2-3001.

(1) Load Condition No. 1 - unusual loading condition - construction.

- (a) Dam structure completed.
- (b) No headwater or tailwater.

(2) Load Condition No. 2 - usual loading condition - normal operating.

(a) Pool elevation at top of closed spillway gates where spillway is gated, and at spillway crest where spillway is ungated.

- (b) Minimum tailwater.
- (c) Uplift.
- (d) Ice and silt pressure, if applicable.

(3) Load Condition No. 3 - unusual loading condition - flood discharge.

- (a) Pool at standard project flood (SPF).
- (b) Gates at appropriate flood-control openings and tailwater at flood elevation.
- (c) Tailwater pressure.
- (d) Uplift.
- (e) Silt, if applicable.
- (f) No ice pressure.

(4) Load Condition No. 4 - extreme loading condition - construction with operating basis earthquake (OBE).

- (a) Operating basis earthquake (OBE).
- (b) Horizontal earthquake acceleration in upstream direction.
- (c) No water in reservoir.
- (d) No headwater or tailwater.

(5) Load Condition No. 5 - unusual loading condition - normal operating with operating basis earthquake.

- (a) Operating basis earthquake (OBE).
- (b) Horizontal earthquake acceleration in downstream direction.
- (c) Usual pool elevation.

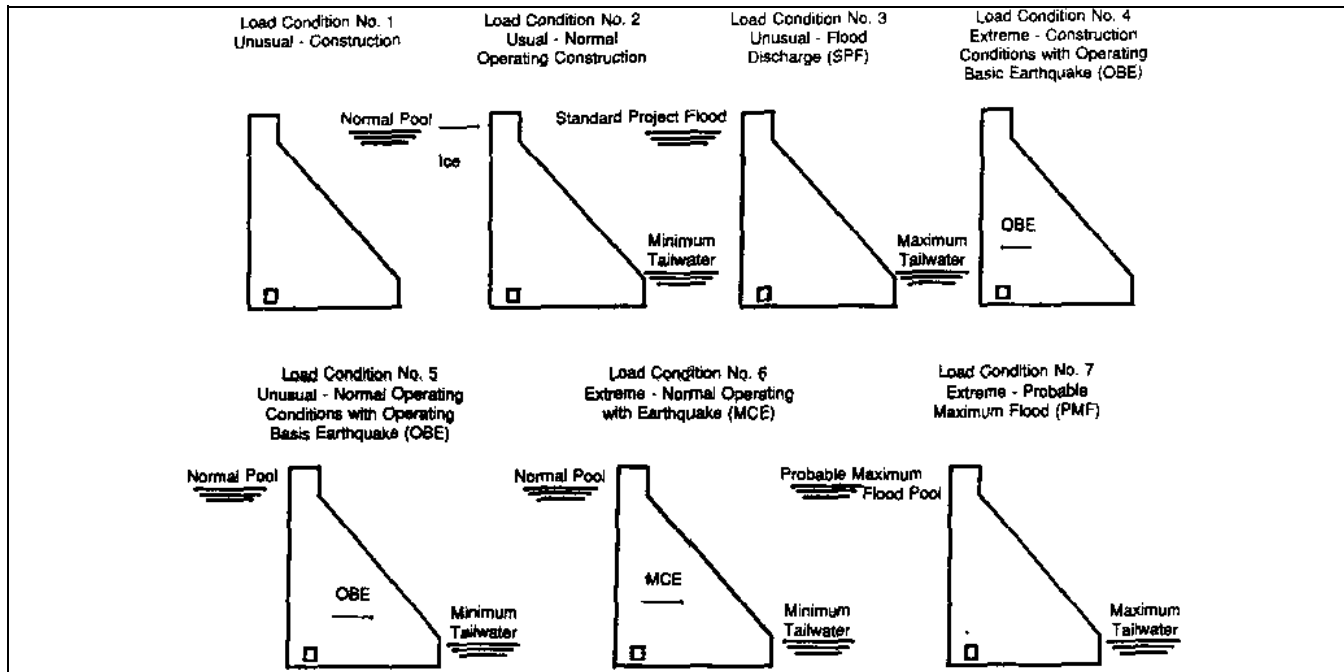


Figure 4-1. Basic loading conditions in concrete gravity dam design

- (d) Minimum tailwater.
 - (e) Uplift at pre-earthquake level.
 - (f) Silt pressure, if applicable.
 - (g) No ice pressure.
- (6) Load Condition No. 6 - extreme loading condition - normal operating with maximum credible earthquake.
- (a) Maximum credible earthquake (MCE).
 - (b) Horizontal earthquake acceleration in downstream direction.
 - (c) Usual pool elevation.
 - (d) Minimum tailwater.
 - (e) Uplift at pre-earthquake level.
 - (f) Silt pressure, if applicable.
 - (g) No ice pressure.
- (7) Load Condition No. 7 - extreme loading condition - probable maximum flood.
- (a) Pool at probable maximum flood (PMF).
 - (b) All gates open and tailwater at flood elevation.
 - (c) Uplift.
 - (d) Tailwater pressure.
 - (e) Silt, if applicable.
 - (f) No ice pressure.
- b.* In Load Condition Nos. 5 and 6, the selected pool elevation should be the one judged likely to exist coincident with the selected design earthquake event. This means that the pool level occurs, on the average, relatively frequently during the course of the year.

4-3. Dam Profiles

a. Nonoverflow section.

- (1) The configuration of the nonoverflow section is usually determined by finding the optimum cross section

that meets the stability and stress criteria for each of the loading conditions. The design cross section is generally established at the maximum height section and then used along the rest of the nonoverflow dam to provide a smooth profile. The upstream face is generally vertical, but may include a batter to increase sliding stability or in existing projects provided to meet prior stability criteria for construction requiring the resultant to fall within the middle third of the base. The downstream face will usually be a uniform slope transitioning to a vertical face near the crest. The slope will usually be in the range of 0.7H to 1V, to 0.8H to 1V, depending on uplift and the seismic zone, to meet the stability requirements.

(2) In the case of RCC dams not using a downstream forming system, it is necessary for construction that the slope not be steeper than 0.8H to 1V and that in applicable locations, it include a sacrificial concrete because of the inability to achieve good compaction at the free edge. The thickness of this sacrificial material will depend on the climatology at the project and the overall durability of the mixture. The weight of this material should not be included in the stability analysis. The upstream face will usually be vertical to facilitate construction of the facing elements. When overstressing of the foundation material becomes critical, constructing a uniform slope at the lower part of the downstream face may be required to reduce foundation pressures. In locations of slope changes, stress concentrations will occur. Stresses should be analyzed in these areas to assure they are within acceptable levels.

(3) The dam crest should have sufficient thickness to resist the impact of floating objects and ice loads and to meet access and roadway requirements. The freeboard at the top of the dam will be determined by wave height and runoff. In significant seismicity areas, additional concrete near the crest of the dam results in stress increases. To reduce these stress concentrations, the crest mass should be kept to a minimum and curved transitions provided at slope changes.

b. Overflow section. The overflow or spillway section should be designed in a similar manner as the non-overflow section, complying with stability and stress criteria. The upstream face of the overflow section will have the same configuration as the nonoverflow section. The required downstream face slope is made tangent to the exponential curve of the crest and to the curve at the junction with the stilling basin or flip bucket. The methods used to determine the spillway crest curves is covered in EM 1110-2-1603, Hydraulic Design of Spillways. Piers may be included in the overflow section

to support a bridge crossing the spillway and to support spillway gates. Regulating outlet conduits and gates are generally constructed in the overflow section.

4-4. Stability Considerations

a. General requirements. The basic stability requirements for a gravity dam for all conditions of loading are:

(1) That it be safe against overturning at any horizontal plane within the structure, at the base, or at a plane below the base.

(2) That it be safe against sliding on any horizontal or near-horizontal plane within the structure at the base or on any rock seam in the foundation.

(3) That the allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

Characteristic locations within the dam in which a stability criteria check should be considered include planes where there are dam section changes and high concentrated loads. Large galleries and openings within the structure and upstream and downstream slope transitions are specific areas for consideration.

b. Stability criteria. The stability criteria for concrete gravity dams for each load condition are listed in Table 4-1. The stability analysis should be presented in the design memoranda in a form similar to that shown on Figure 4-1. The seismic coefficient method of analysis, as outlined in Chapter 3, should be used to determine resultant location and sliding stability for the earthquake load conditions. The seismic coefficient used in the analysis should be no less than that given in ER 1110-2-1806, Earthquake Design and Analysis for Corps of Engineers Projects. Stress analyses for a maximum credible earthquake event are covered in Chapter 5. Any deviation from the criteria in Table 4-1 shall be accomplished only with the approval of CECW-ED, and should be justified by comprehensive foundation studies of such nature as to reduce uncertainties to a minimum.

4-5. Overturning Stability

a. Resultant location. The overturning stability is calculated by applying all the vertical forces (ΣV) and lateral forces for each loading condition to the dam and, then, summing moments (ΣM) caused by the consequent forces about the downstream toe. The resultant location along the base is:

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	\leq allowable	$0.3 f'_c$	0
Unusual	Middle 1/2	1.7	\leq allowable	$0.5 f'_c$	$0.6 f'_c{}^{2/3}$
Extreme	Within base	1.3	$\leq 1.33 \times$ allowable	$0.9 f'_c$	$1.5 f'_c{}^{2/3}$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (FS) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

$$Resultant\ location = \frac{\sum M}{\sum V} \quad (4-1)$$

$$FS = \frac{\tau_F}{\tau} = \frac{(\sigma \tan \phi + c)}{\tau} \quad (4-2)$$

The methods for determining the lateral, vertical, and uplift forces are described in Chapter 3.

b. Criteria. When the resultant of all forces acting above any horizontal plane through a dam intersects that plane outside the middle third, a noncompression zone will result. The relationship between the base area in compression and the location of the resultant is shown in Figure 4-2. For usual loading conditions, it is generally required that the resultant along the plane of study remain within the middle third to maintain compressive stresses in the concrete. For unusual loading conditions, the resultant must remain within the middle half of the base. For the extreme load conditions, the resultant must remain sufficiently within the base to assure that base pressures are within prescribed limits.

4-6. Sliding Stability

a. General. The sliding stability is based on a factor of safety (FS) as a measure of determining the resistance of the structure against sliding. The multiple-wedge analysis is used for analyzing sliding along the base and within the foundation. For sliding of any surface within the structure and single planes of the base, the analysis will follow the single plane failure surface of analysis covered in paragraph 4-6e.

b. Definition of sliding factor of safety.

(1) The sliding FS is conceptually related to failure, the ratio of the shear strength (τ_F), and the applied shear stress (τ) along the failure planes of a test specimen according to Equation 4-2:

where $\tau_F = \sigma \tan \phi + c$, according to the Mohr-Coulomb Failure Criterion (Figure 4-3). The sliding FS is applied to the material strength parameters in a manner that places the forces acting on the structure and rock wedges in sliding equilibrium.

(2) The sliding FS is defined as the ratio of the maximum resisting shear (T_F) and the applied shear (T) along the slip plane at service conditions:

$$FS = \frac{T_F}{T} = \frac{(N \tan \phi + cL)}{T} \quad (4-3)$$

where

N = resultant of forces normal to the assumed sliding plane

ϕ = angle of internal friction

c = cohesion intercept

L = length of base in compression for a unit strip of dam

c. Basic concepts, assumptions, and simplifications.

(1) Limit equilibrium. Sliding stability is based on a limit equilibrium method. By this method, the shear force necessary to develop sliding equilibrium is determined for an assumed failure surface. A sliding mode of failure will occur along the presumed failure surface when the applied shear (T) exceeds the resisting shear (T_F).

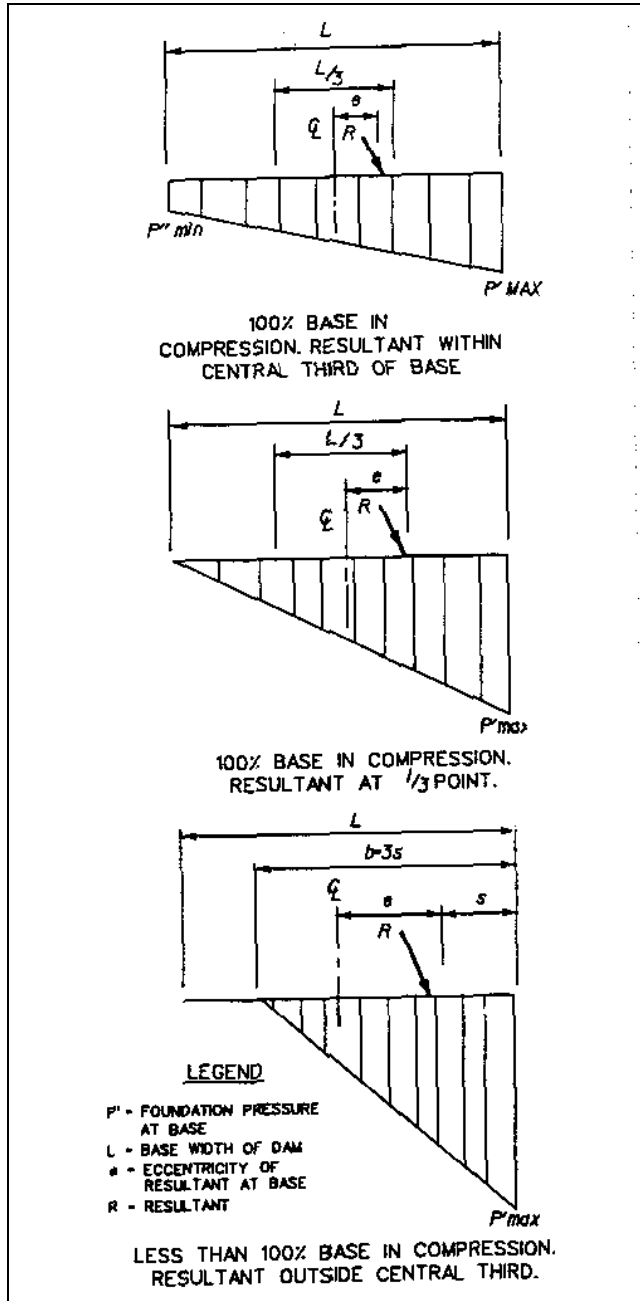


Figure 4-2. Relationship between base area in compression and resultant location

(2) Failure surface. The analyses are based on failure surfaces that can be any combination of planes and curves; however, for simplicity all failure surfaces are assumed to be planes. These planes form the bases of the wedges. It should be noted that for the analysis to be realistic, the assumed failure planes have to be kinematically possible. In rock the slip planes may be

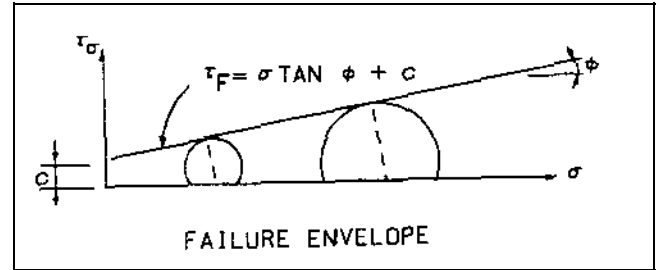


Figure 4-3. Failure envelope

predetermined by discontinuities in the foundation. All the potential planes of failure must be defined and analyzed to determine the one with the least FS .

(3) Two-dimensional analysis. The principles presented for sliding stability are based on a two-dimensional analysis. These principles should be extended to a three-dimensional analysis if unique three-dimensional geometric features and loads critically affect the sliding stability of a specific structure.

(4) Force equilibrium only. Only force equilibrium is satisfied in the analysis. Moment equilibrium is not used. The shearing force acting parallel to the interface of any two wedges is assumed to be negligible; therefore, the portion of the failure surface at the bottom of each wedge is loaded only by the forces directly above or below it. There is no interaction of vertical effects between the wedges. The resulting wedge forces are assumed horizontal.

(5) Displacements. Considerations regarding displacements are excluded from the limit equilibrium approach. The relative rigidity of different foundation materials and the concrete structure may influence the results of the sliding stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit-equilibrium approach. The effects of strain compatibility along the assumed failure surface may be approximated in the limit-equilibrium approach by selecting the shear strength parameters from in situ or laboratory tests according to the failure strain selected for the stiffest material.

(6) Relationship between shearing and normal forces. A linear relationship is assumed between the resisting shearing force and the normal force acting on the slip plane beneath each wedge. The Coulomb-Mohr Failure Criterion defines this relationship.

d. Multiple wedge analysis.

(1) General. This method computes the sliding FS required to bring the sliding mass, consisting of the structural wedge and the driving and resisting wedges, into a state of horizontal equilibrium along a given set of slip planes.

(2) Analysis model. In the sliding stability analysis, the gravity dam and the rock and soil acting on the dam are assumed to act as a system of wedges. The dam foundation system is divided into one or more driving wedges, one structural wedge, and one or more resisting wedges, as shown in Figures 4-4 and 4-5.

(3) General wedge equation. By writing equilibrium equations normal and parallel to the slip plane, solving for N_i and T_i , and substituting the expressions for N_i and T_i into the equation for the factor of safety of the typical

wedge, the general wedge and wedge interaction equation can be written as shown in Equation 4-5 (derivation is provided in Appendix C).

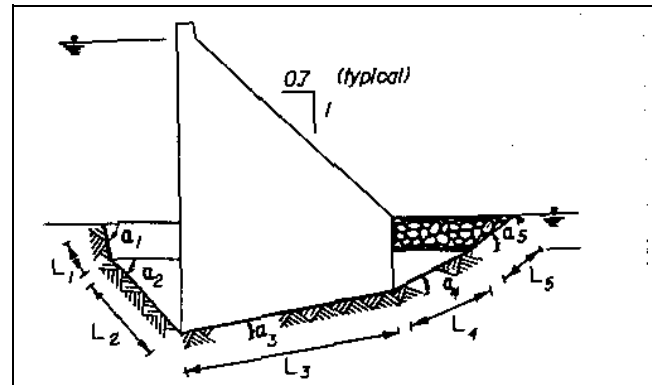


Figure 4-4. Geometry of structure foundation system

$$FS = \frac{\left[(W_i + V_i) \cos \alpha_i + (H_{Li} - H_{Ri}) \sin \alpha_i + (P_{i-1} - P_i) \sin \alpha_i - U_i \right] \tan \phi_i + CL_i}{\left[(H_{Li} - H_{Ri}) \cos \alpha_i + (P_{i-1} - P_i) \cos \alpha_i - (W_i + V_i) \sin \alpha_i \right]} \quad (4-5)$$

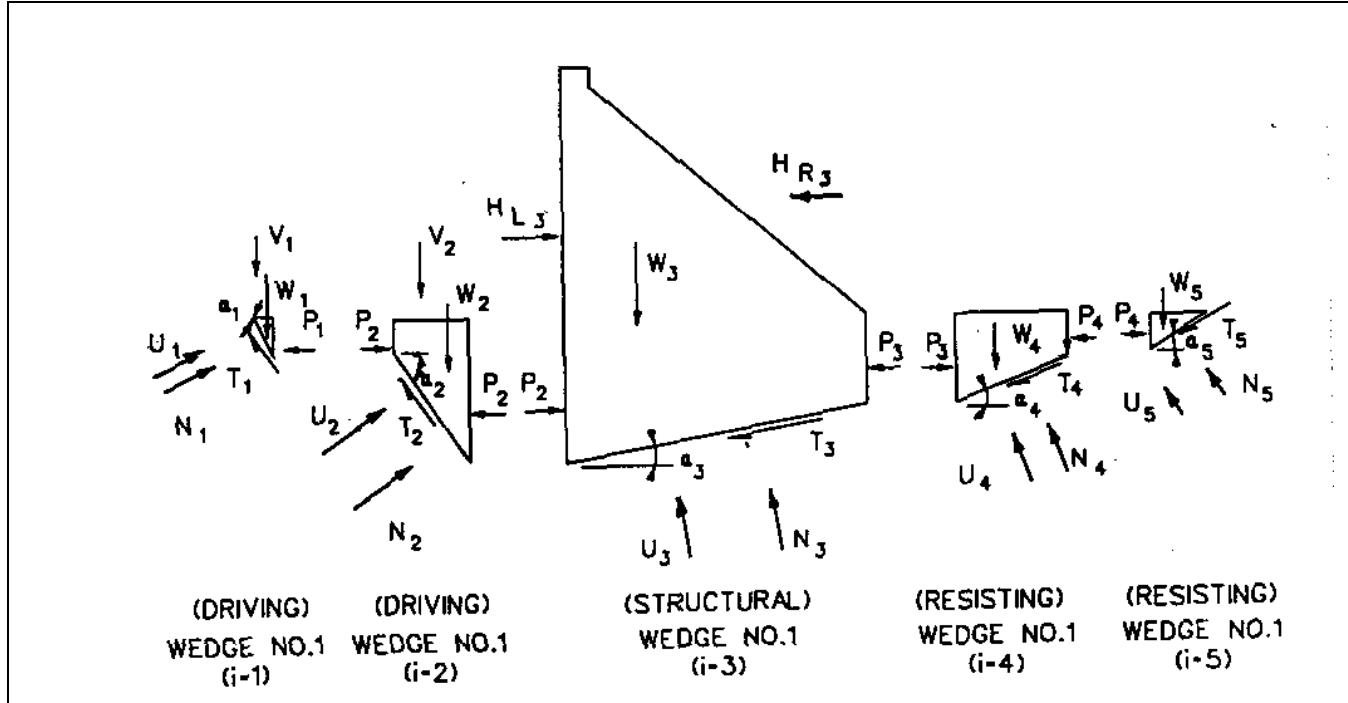


Figure 4-5. Dam foundation system, showing driving, structural, and resisting wedges

Solving for $(P_{i-1} - P_i)$ gives the general wedge equation,

$$(P_{i-1} - P_i) = \left[(W_i + V_i)(\tan \phi_{di} \cos \alpha_i + \sin \alpha_i) - U_i \tan \phi_{di} (H_{Li} - H_{Ri}) \right. \\ \left. (\tan \phi_{di} \sin \alpha_i - \cos \alpha_i) + c_{di} L_i \right] / (\cos \alpha_i - \tan \phi_{di} \sin \alpha_i) \quad (4-6)$$

where

i = number of wedge being analyzed

$(P_{i-1} - P_i)$ = summation of applied forces acting horizontally on the i^{th} wedge. (A negative value for this term indicates that the applied forces acting on the i^{th} wedge exceed the forces resisting sliding along the base of the wedge. A positive value for the term indicates that the applied forces acting on the i^{th} wedge are less than the forces resisting sliding along the base of that wedge.)

W_i = total weight of water, soil, rock, or concrete in the i^{th} wedge

V_i = any vertical force applied above top of i^{th} wedge

$\tan \phi_{di} = \tan \phi_i / FS$

α_i = angle between slip plane of i^{th} wedge and horizontal. Positive is counterclockwise

U_i = uplift force exerted along slip plane of the i^{th} wedge

H_{Li} = any horizontal force applied above top or below bottom of left side adjacent wedge

H_{Ri} = any horizontal force applied above top or below bottom of right side adjacent wedge

$c_{di} = c_i / FS$

L_i = length along the slip plane of the i^{th} wedge

This equation is used to compute the sum of the applied forces acting horizontally on each wedge for an assumed FS . The same FS is used for each wedge. The derivation of the general wedge equation is covered in Appendix C.

(4) Failure plane angle. For the initial trial, the failure plane angle α for a driving wedge can be approximated by:

$$\alpha = 45^\circ + \frac{\phi_d}{2}$$

$$\text{where } \phi_d = \tan^{-1} \left(\frac{\tan \phi}{FS} \right)$$

For a resisting wedge, the slip plane angle can be approximated by:

$$\alpha = 45^\circ - \frac{\phi_d}{2}$$

These equations for the slip plane angle are the exact solutions for wedges with a horizontal top surface with or without a uniform surcharge.

(5) Procedure for a multiple-wedge analysis. The general procedure for analyzing multi-wedge systems includes:

(a) Assuming a potential failure surface based on the stratification, location and orientation, frequency and distribution of discontinuities of the foundation material, and the configuration of the base.

(b) Dividing the assumed slide mass into a number of wedges, including a single-structure wedge.

(c) Drawing free body diagrams that show all the forces assuming to be acting on each wedge.

(d) Estimate the FS for the first trial.

(e) Compute the critical sliding angles for each wedge. For a driving wedge, the critical angle is the

angle that produces a maximum driving force. For a resisting wedge, the critical angle is the angle that produces a minimum resisting force.

(f) Compute the uplift pressure, if any, along the slip plane. The effects of seepage and foundation drains should be included.

(g) Compute the weight of each wedge, including any water and surcharges.

(h) Compute the summation of the lateral forces for each wedge using the general wedge equation. In certain cases where the loadings or wedge geometries are complicated, the critical angles of the wedges may not be easily calculated. The general wedge equation may be used to iterate and find the critical angle of a wedge by varying the angle of the wedge to find a minimum resisting or maximum driving force.

(i) Sum the lateral forces for all the wedges.

(j) If the sum of the lateral forces is negative, decrease the FS and then recompute the sum of the lateral forces. By decreasing the FS , a greater percentage of the shearing strength along the slip planes is mobilized. If the sum of the lateral forces is positive, increase the FS and recompute the sum of the lateral forces. By increasing the FS , a smaller percentage of the shearing strength is mobilized.

(k) Continue this trial and error process until the sum of the lateral forces is approximately zero for the FS used. This procedure will determine the FS that causes the sliding mass in horizontal equilibrium, in which the sum of the driving forces acting horizontally equals the sum of the resisting forces that act horizontally.

(l) If the FS is less than the minimum criteria, a redesign will be required by sloping or widening the base.

e. Single-plane failure surface. The general wedge equation reduces to Equation 4-7 providing a direct solution for FS for sliding of any plane within the dam and for structures defined by a single plane at the interface between the structure and foundation material with no embedment. Figure 4-6 shows a graphical representation of a single-plane failure mode for sloping and horizontal surfaces.

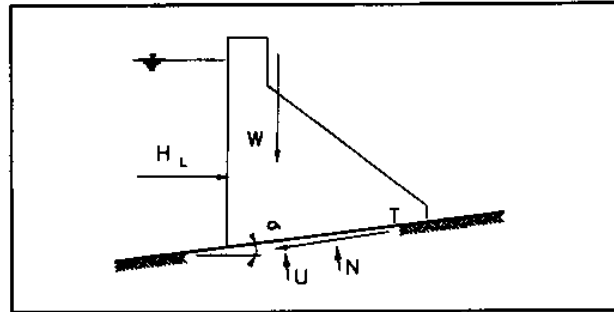
$$FS = \frac{[W \cos \alpha - U + H \sin \alpha] \tan \phi + CL}{H \cos \alpha - W \sin \alpha} \quad (4-7)$$

where

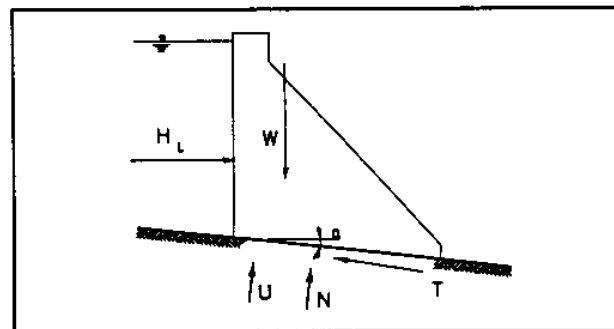
H = horizontal force applied to dam

C = cohesion on slip plane

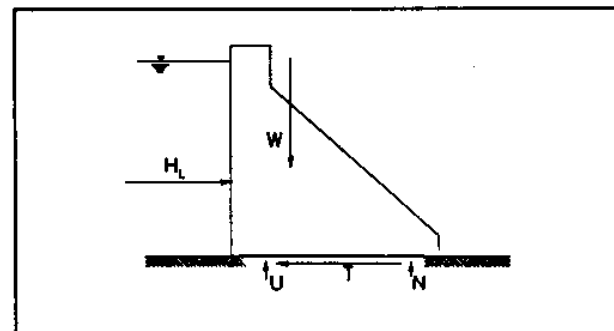
L = length along slip plane



a. Upslope sliding, $\alpha > 0$



b. Downslope slid



c. Horizontal sliding, $\alpha = 0$

Figure 4-6. Single plane failure mode

For the case of sliding through horizontal planes, generally the condition analyzed within the dam, Equation 4-7 reduces to Equation 4-8:

$$FS = \frac{(W - U) \tan \phi + CL}{H_L} \quad (4-8)$$

f. Design considerations.

(1) Driving wedges. The interface between the group of driving wedges and the structural wedge is assumed to be a vertical plane that is located at the heel of the structural wedge and extends to its base. The magnitudes of the driving forces depend on the actual values of the safety factor and the inclination angles of the slip path. The inclination angles, corresponding to the maximum active forces for each potential failure surface, can be determined by independently analyzing the group of driving wedges for a trial safety factor. In rock, the inclination may be predetermined by discontinuities in the foundation. The general equation applies directly only to driving wedges with assumed horizontal driving forces.

(2) Structural wedge. The general wedge equation is based on the assumption that shearing forces do not act on the vertical wedge boundaries; hence there can be only one *structural wedge* because concrete structures transmit significant shearing forces across vertical internal planes. Discontinuities in the slip path beneath the structural wedge should be modeled by assuming an average slip plane along the base of the structural wedge.

(3) Resisting wedges. The interface between the group of *resisting wedges* and the structural wedge is assumed to be a vertical plane that is located at the toe of the structural wedge and extends to its base. The magnitudes of the resisting forces depend on the actual values of the safety factor and the inclination angles of the slip path. The inclination angles, corresponding to the minimum passive forces for each potential failure mechanism, can be determined by independently analyzing the group of resisting wedges for a trial safety factor. The general wedge equation applies directly only to resisting wedges with assumed horizontal passive forces. If passive resistance is used, then rock that may be subjected to high velocity water scouring should not be used unless adequately protected. Also, the compressive strength of the rock layers must be sufficient to develop the wedge resistance. In some cases, wedge resistance should not be included unless rock anchors are installed to stabilize the wedge.

(4) Effects of cracks in foundation. Sliding analyses should consider the effects of cracks on the driving side of the structural wedge in the foundation material resulting from differential settlement, shrinkage, or joints in a rock mass. The depth of cracking in massive strong rock foundations should be assumed to extend to the base of the structural wedge. Shearing resistance along the crack should be ignored, and full hydrostatic pressure should be assumed to act at the bottom of the crack. The hydraulic gradient across the base of the structural wedge should reflect the presence of a crack at the heel of the structural wedge.

(5) Uplift. The effects of uplift forces should be included in the sliding analysis. Uplift pressures on the wedges and within any plane within the structure should be determined as described in Chapter 3, Section 3.

(6) Resultant outside kern. As previously stated, requirements for rotational equilibrium are not directly included in the general wedge equation. For some load cases, the normal component of the resultant applied loads will lie outside the kern of the base area, and not all of the structural wedge will be in contact with the foundation material. The sliding analysis should be modified for these load cases to reflect the following secondary effects due to *coupling of the sliding and rotational behavior*.

(a) The uplift pressure on the portion of the base not in contact with the foundation material should be a uniform value that is equal to the maximum value of the hydraulic pressure across the base (except for instantaneous load cases such as those resulting from seismic forces).

(b) The cohesive component of the sliding resistance should include only the portion of the base area in contact with the foundation material.

(7) Seismic sliding stability. The sliding stability of a structure for an earthquake-induced base motion should be checked by assuming the specified horizontal earthquake and the vertical earthquake acceleration, if included in the analysis, to act in the most unfavorable direction. The earthquake-induced forces on the structure and foundation wedges may then be determined by the seismic coefficient method as outlined in Chapter 3. Lateral earthquake forces for resisting and driving wedges consisting of soil material should be determined as described in EM 1110-2-2502, Retaining and Flood Walls.

(8) Strain compatibility. Shear resistance in a dam foundation is dependent on the strength properties of the rock. Slide planes within the foundation rock may pass through different materials, and these surfaces may be either through intact rock or along existing rock discontinuities. Less deformation is required for intact rock to reach its maximum shear resistance than for discontinuity surfaces to develop their maximum frictional resistances. Thus, the shear resistance developed along discontinuities depends on the amount of displacement on the intact rock part of the shear surface. If the intact rock breaks, the shear resistance along the entire length of the shear plane is the combined frictional resistance for all materials along the plane.

4-7. Base Pressures

a. Computations of base pressures. For the dam to be in static equilibrium, the resultant of all horizontal and vertical forces including uplift must be balanced by an equal and opposite reaction of the foundation consisting of the total normal reaction and the total tangential shear. The location of this force is such that the summation of moments is equal to zero.

b. Allowable base pressure. The maximum computed base pressure should be equal to or less than the allowable bearing capacity for the usual and unusual load conditions. For extreme loading condition, the maximum bearing pressure should be equal to or less than 1.33 times the allowable bearing capacity.

4-8. Computer Programs

a. Program for sliding stability analysis of concrete structures (CSLIDE).

(1) The computer program CSLIDE has the capability of performing a two-dimensional sliding stability analysis of gravity dams and other concrete structures. It uses the principles of the multi-wedge system of analysis as discussed in paragraph 4-6. Program documentation is covered in U.S. Army Engineer Waterways Experiment Station (WES) Instruction Report ITL-87-5, "Sliding Stability of Concrete Structures (CSLIDE)."

(2) The potential failure planes and the associated wedges are chosen for input and, by satisfying limit equilibrium principles, the *FS* against sliding failure is computed for output. The results also give a summary of failure angles and forces acting on the wedges.

(3) The program considers the effects of:

- (a) Multiple layers of rock with irregular surfaces.
 - (b) Water and seepage effects. The line-of-creep and seepage factor/gradient are provided.
 - (c) Applied vertical surcharge loads including line, uniform, strip, triangular, and ramp loads.
 - (d) Applied horizontal concentrated point loads.
 - (e) Irregularly shaped structural geometry with a horizontal or sloped base.
 - (f) Percentage of the structure base in compression because of overturning effects.
 - (g) Single and multiple-plane options for the failure surfaces.
 - (h) Horizontal and vertical induced loads because of earthquake accelerations.
 - (i) Factors requiring the user to predetermine the failure surface.
- (4) It will not analyze curved surfaces or discontinuities in the slip surface of each wedge. In those cases, an average linear geometry should be assumed along the base of the wedge.

b. Three-dimensional stability analysis and design program (3DSAD), special purpose modules for dams (CDAMS).

(1) General. The computer program called CDAMS performs a three-dimensional stability analysis and design of concrete dams. The program was developed as a specific structure implementation of the three-dimensional stability analysis and design (3DSAD) program. It is intended to handle two cross-sectional types:

- (a) An overflow monolith with optional pier.
- (b) A nonoverflow monolith.

The program can operate in either an analysis or design mode. Load conditions outlined in paragraph 4-1 can be performed in any order. A more detailed description and information about the use of the program can be found in

Instruction Report K-80-4, "A Three-Dimensional Stability Analysis/Design Program (3DSAD); Report 4, Special Purpose Modules for Dams (CDAMS)" (U.S. Army Corps of Engineers (USACE) 1983).

(2) Analysis. In the analysis mode, the program is capable of performing resultant location, bearing, and sliding computations for each load condition. A review is made of the established criteria and the results outputted.

(3) Design. In the design mode, the structure is incrementally modified until a geometry is established that meets criteria. Different geometric parameters may be varied to achieve a stable geometry. A design memorandum plate option is also available.